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## Chapter Sixty-seven

# BENTS, PIERS, AND ABUTMENTS

References shown following section titles are to the AASHTO LRFD Bridge Design Specifications.

Section 11 of the *LRFD Bridge Design Specifications* discusses the design requirements of bents, piers, and abutments. This Chapter describes supplementary information on the design of these structural components. See Section 59-2.0 for more information on substructure types and their selection.

### **67-1.0 SPILL-THROUGH END BENTS**

Spill-through end bents are the most common form of bridge end support treatment presently used. A spill-through end bent may be designed as one of two types: integral end bent (without joint between substructure and superstructure), or non-integral end bent (with joint). These are discussed in this Section. See Section 59-2.0 for more information on spill-through end bents.

#### **67-1.01 Integral End Bents**

##### **67-1.01(01) General**

Traditionally, bridges have been designed with expansion joints and other structural releases that allow the superstructure to expand and contract relatively freely with changing temperatures and other geometric effects. Integral end bents eliminate expansion joints in the bridge deck, which reduce both the initial construction costs and subsequent maintenance costs.

The use of integral end bents is very effective in accommodating the horizontal seismic forces of Seismic Performance Zones 1 and 2. Minimum support length requirements need not be investigated for an integral end bent bridge.

##### **67-1.01(02) Usage (New Structure)**

Integral end bents should be used for a new structure in accordance with the geometric limitations provided in Figure 67-1A.

### **67-1.01(03) Usage (Existing Structure)**

For an existing bridge without integral end bents, the design criteria in Figure 67-1A should be used when evaluating the conversion to an integral end bent structure. For additional information, see Section 72-3.04.

### **67-1.01(04) General Design Criteria**

The following requirements must be satisfied.

1. Backfill. Each integral end bent for a beam/girder type superstructure shall be backfilled with coarse aggregate, under the pay item, aggregate for end bent backfill. Each reinforced concrete slab bridge end bent shall be backfilled with flowable backfill material. The INDOT *Standard Drawings* present backfill details for both concrete slab and beam/girder type structures.

The total estimated quantity of flowable backfill or aggregate for end bent backfill shall be shown on the Layout Sheet of the plans.

2. Bridge Approach. A reinforced concrete bridge approach, anchored to the end bent with epoxy coated #16 bars spaced at 300-mm centers, shall be used at each integral end bent regardless of the traffic volume. These bars shall extend out of the pavement ledge as shown in Figures 67-1B and 67-1C. Two layers of 150-µm minimum polyethylene sheeting shall be placed between the reinforced concrete bridge approach and the subgrade. A rigid reinforced concrete bridge approach is necessary to prevent compaction of the backfill behind the end bent.
3. Bridge Approach Joint. A 600-mm wide terminal joint or pavement relief joint shall be used at the roadway end of the reinforced concrete bridge approach if any portion of the adjacent pavement section is concrete. A joint is not required if the entire adjacent pavement section is asphalt.
4. Wingwall Configuration. Wingwalls should extend parallel to the centerline of roadway. This configuration reduces the loads imposed upon the bridge structure due to passive earth pressure from the end bent backfill.
5. Wingwall Connection. The connection between the wingwall and the end bent cap should be treated as described below. The wingwall should not extend more than 3 m behind the rear face of the cap. If longer extensions are necessary, force effects in the

connection between the wingwall and cap, and in the wingwall itself, shall be investigated and adequate reinforcing steel should be provided.

6. Interior Diaphragms for Steel Structure. Where steel beams or girders are used, an interior diaphragm should be placed within 3 m of the end support to provide beam stability prior to and during the deck pour.

#### **67-1.01(05) Superstructure and Interior Substructure Design Criteria**

Although the ends of the superstructure are monolithically attached to integral end bents, the rotation permitted by the piles is sufficiently high, and the attendant end moment sufficiently low, to justify the assumption of a pinned-end condition for design. The following design assumptions shall be considered.

1. Ends. The ends of the superstructure are free to rotate and translate longitudinally.
2. Passive Earth Pressure. The restraining effect of passive earth pressure behind the end bents shall be neglected when considering superstructure longitudinal force distribution to the interior piers.
3. Interior Pile Bents. All longitudinal forces from the superstructure are generally disregarded when designing an interior pile bent or a thin wall pier on a single row of piles.
4. Shears/Moments. Force effects in the cap beam may be determined on the basis of a linear distribution of vertical pile reactions. For minimum reinforcement, the cap should be treated as a structural beam.

#### **67-1.01(06) End Bent Details**

Integral end bents may be constructed using either of the following methods.

1. Method A. The superstructure beams are placed on and attached directly to the end bent piling. The entire end bent is then poured at the same time as the superstructure deck. This is the preferred method.
2. Method B. The superstructure beams are set in place and anchored to the previously cast-in-place end bent cap. The concrete above the previously cast-in-place cap shall be poured at the same time as the superstructure deck.

Optional construction joints may be placed in the end bent cap to facilitate construction. The optional joint below the bottom of beam may be used regardless of bridge length. To accommodate the contractor's common practice of pouring the reinforced concrete bridge approach with the bridge deck, a wrap should be provided on the top portion of the pile encased in the concrete cap to counteract any moments that may be prematurely induced at the top of the pile by this practice. The wrap should consist of 25-mm expanded polystyrene. The wrap should not be used with a reinforced concrete slab bridge. The optional construction joint at the pavement ledge elevation shown in Figures 67-1B and 67-1C allows the contractor to pour the reinforced concrete bridge approach with the bridge deck.

Regardless of the method used, the end bent details should be in accordance with the following:

1. Width. The end bent width shall not be less than 750 mm.
2. Cap Embedment. The piling shall extend a minimum of 600 mm into the cap if using Method A, and a minimum of 375 mm if using Method B.
3. Beam Attachment. The beams shall be physically attached to the end bent piling if using Method A and to the cast-in-place cap if using Method B.
4. Beam Extension. The beams shall extend at least 500 mm into the bent measured along the centerline of the beam.
5. Concrete Cover. Concrete cover beyond the farthest most edge of the beam at the rear face of the bent shall be at least 100 mm. This minimum cover shall also apply to the pavement ledge area. The top flanges of steel beams and prestressed I-beams may be coped to meet this requirement. Where the 100-mm minimum cover cannot be maintained within a 750-mm cap, the cap shall be widened.
6. Stiffener Plates. Steel beams and girders shall have 15-mm stiffener plates welded to both sides of the web and to the flanges over the supports to anchor the beams into the concrete. In addition, a minimum of three holes shall be provided through the webs of steel beams and girders. Two holes should be provided through prestressed I-beam webs near the front face of the bent, to allow #19 bars to be inserted to further anchor the beam to the cap. Box beams shall have two threaded inserts placed in each side face for anchorage of #22 threaded bars.
7. Reinforcement. The minimum size of stirrups shall be #19 spaced at a maximum of 300 mm. Longitudinal cap reinforcing shall be #22 at 300-mm maximum spacing along both faces of the bent. All reinforcing steel shall be epoxy coated.

8. Corner Bars. Corner bars should extend from the rear face of the cap into the top of the deck at not more than 300-mm spacing as shown in Figures 67-1B and 67-1C.

### **67-1.01(07) Plan Details**

Section 62-3.0 includes design details for integral end bents with a reinforced concrete slab bridge. Figures 67-1B and 67-1C show suggested details for integral end bents with a girder bridge. Other reinforcing and connection details should be considered and used where they are structurally sound and afford a definite advantage when compared to those shown in the Figures.

### **67-1.02 Non-Integral End Bents**

#### **67-1.02(01) Usage**

Non-integral end bents are used where integral end bents are not appropriate. They shall be backfilled with coarse aggregate material, under the pay item, aggregate for end bent backfill. The INDOT *Standard Drawings* show backfill details for beam/girder type structures.

#### **67-1.02(02) Mudwalls**

A mudwall must be 300 mm wide with minimum vertical reinforcing steel of #13 at 300 mm. For a mudwall greater than 1500 mm in height and for a mudwall with large joint forces, the designer should investigate the mudwall reinforcement.

#### **67-1.02(03) Epoxy Coated Reinforcing Steel**

For an end bent with a bridge deck expansion joint located between the end of the deck and the face of the mudwall, all reinforcing steel in the end bent shall be epoxy coated. This includes all cap, mudwall and, if present, wingwall reinforcing.

#### **67-1.02(04) Connection to Reinforced Concrete Bridge Approach**

The reinforced concrete bridge approach shall be connected to the adjacent mudwall with epoxy coated #16 bars spaced at 300-mm centers. These bars shall extend out of the pavement ledge as shown in Figure 67-1D. For details of the connection of a reinforced concrete bridge approach carried over the top of a mudwall, see Figure 67-1E.

### **67-1.02(05) Beam and Mudwall Slots**

Beam slots (concrete channels) may be formed into the top of the end bent cap to provide lateral restraint for beams which do not have side restraint provided by the bearings or other means. The vertical face of the slot shall extend a minimum of 50 mm above the bottom of beam or bearing plate. A height of 150 mm will often suffice. If a concrete deck is carried over the top of a mudwall, channel slots may be provided in the top of the mudwall to provide lateral restraint for the superstructure. The latter technique is generally used in a bridge rehabilitation project, and may be used for a new bridge that exceeds the skew limits, but not the length limits, for integral end bents.

Figure 67-1F presents a schematic of beam slots at a non-integral end bent with a deck expansion joint. Figure 67-1G presents a schematic of mudwall slots at a jointless, non-integral end bent.

### **67-1.02(06) Design Details**

Figure 67-1F presents design details for a pile cap with minimum cap thickness for a jointed, non-integral end bent. Figure 67-1G presents design details for a pile cap with minimum cap thickness for a jointless, non-integral end bent.

## **67-1.03 Pile Spacings and Loads**

### **67-1.03(01) General Design Criteria**

The following criteria applies to piling for both integral and non-integral end bents.

1. Pile Spacing. Pile spacing should not normally exceed 3.0 m. If the cap is properly analyzed and designed as a continuous beam, this restriction need not apply. If practical, one pile may be placed beneath each girder. To reduce force effects for a large beam spacing, consideration may be given to twin piles under the beam, spaced at not less than 750 mm. See Chapter Sixty-six for minimum pile spacings. For an integral end bent within the limits defined in Figure 67-1A and for a non-integral end bent, the piles are considered to be free ended and capable of resisting only horizontal and vertical forces.
2. Number. The number of piles per support shall not be less than four. Three piles per support may be used in a local-public-agency structure if the agency has documented its approval of a three-beam/girder superstructure.
3. Overhang. The minimum cap overhang shall be 450 mm.

4. Pile Overload. If an individual pile is overloaded due to the maximum beam or girder loads, the overload amount may be considered equally distributed to the two adjacent piles provided that this distribution of overloads does not cause either of the adjacent piles to exceed their allowable bearing capacity. This distribution of overload will only be allowed if the allowable bearing value for the pile is based upon the capacity of the soils and not on the structural strength of the pile, and if the pile cap has enough beam strength to distribute this overload to the adjacent piles.

#### **67-1.03(02) Live Load Distribution**

The wheel loads located out in the span shall be distributed to the substructure in accordance with the live-load distribution factors in Article 4.6.2.2.2 of the *LRFD Specifications*. For wheels located over the support, a simple-span transverse distribution should be used.

#### **67-1.03(03) Integral End Bent**

The following criteria apply specifically to piles and loads at an integral end bent.

1. Loads/Forces. Only vertical loads may be considered when designing end bent piling for a structure which satisfies the requirements provided in Figure 67-1A. Force effects in the end bent piles due to temperature, shrinkage, creep, and horizontal earth pressures may be neglected.

An alternative analysis must be used if the criteria in Figure 67-1A are not met. The following steps should be considered in this analysis.

- a. The point of zero movement should be established by considering the elastic resistance of all substructure elements and bearing devices.
- b. The effects of creep, shrinkage, and temperature should be considered.
- c. Any movement at any point on the superstructure should be taken as being proportional to its distance to the point of zero movement.
- d. Lateral curvature of the superstructure may be neglected if it satisfies Article 4.6.1.2 of the *LRFD Bridge Design Specifications*.
- e. Vertical force effects in the end bent piles should be distributed linearly with load eccentricities properly accounted for.
- f. Lateral soil resistance should be considered in establishing force effects and buckling resistance of piles.
- g. Force effects should be combined in accordance with Article 3.4.1 of the *LRFD Specifications*.

2. Pile Type. Only steel H-piles or steel-encased concrete piles should be used with an integral end bent. Steel H-pile webs shall be placed perpendicular to the centerline of the structure to minimize flexural forces in the piling. All end bent piling shall be driven vertically. Only one row of piling is permitted.
3. Hard Soils. Where an existing cohesive earth stratum, with a standard penetration resistance (N) exceeding 35 blows per foot, is located within the 3-m interval below the bottom of the cap, the pile shall be placed in an oversized predrilled hole before driving. The predrilled hole shall extend 2.5 m below the bottom of the cap. The minimum diameter of the oversized hole shall be 100 mm greater than the maximum cross sectional dimension of the pile. The hole shall be backfilled with uncrushed coarse aggregate size No. 12 following the pile driving operation.

#### **67-1.03(04) Non-Integral End Bent**

The following criteria apply to piles at a non-integral end bent.

1. Pile Spacing. The minimum pile spacing is 750 mm parallel to the centerline of the bent. For a structure with deep girders, two rows of piles with a staggered pile spacing should be considered.
2. Batter. Up to one-half of the piles may be battered to increase the overturning stability of the structure.
3. Overturning. If the pile spacing is less than 3.0 m and one-half of the piles are battered, no investigation is needed for overturning. If less than one-half of the piles are battered and/or if pile spacing is 3.0 m or greater, the stability due to overturning pressures shall be investigated.

#### **67-1.04 Wingwalls**

With respect to spill-through end bents, the following applies to wingwalls.

1. Usage. Each steel or prestressed-concrete beam bridge require wingwalls. A reinforced concrete slab bridge usually does not require wingwalls.
2. Dimensions. Wingwalls shall be of sufficient length and depth to prevent the roadway embankment from encroaching onto the stream channel or clear opening. The slope of the fill will not be steeper than 2:1 (H:V), and wingwall lengths can be established on this

basis. See Figure 67-1H for suggested wingwall dimensioning details. The minimum thickness of any wingwall used with a spill-through end bent shall be 300 mm.

3. Pile-Supported. If wingwalls for a non-integral end bent have a total length of more than 4.5 m, pile support should be investigated. Pile-supported wings should not be used with an integral end bent.
4. Design. Each non-pile-supported wingwall shall be designed as a horizontal cantilevered wall. Because the wingwalls are rigidly attached to the remainder of the bent, the bent is restrained from deflecting except laterally as a unit. Due to the lack of the usual retaining structure rotation, the active soil pressure condition cannot develop, and the design soil pressure must be increased to a value between the active and at-rest condition. Therefore, the horizontal earth pressure to be used in design shall be equal to 150 percent of the value determined assuming an active soil condition. Live-load surcharge should be added to soil loads in accordance with Article 3.11.6.2 of the *LRFD Bridge Design Specifications*.

#### **67-1.05 Drainage**

See the INDOT *Standard Drawings* for details of drainage-pipes placement behind an end bent.

#### **67-1.06 Joints**

##### **67-1.06(01) Construction Joints**

The following applies to construction joints at spill-through end bents.

1. Type. Use construction joint type A for all horizontal construction joints. See the INDOT *Standard Drawings*.
2. Integral. See Figures 67-1B and 67-1C for construction joints at an integral end bent.
3. Non-Integral. See Figures 67-1F and 67-1G for construction joints at a non-integral end bent. A construction joint cannot be placed higher than the pavement ledge.

##### **67-1.06(02) Longitudinal Open Joints**

If the bridge deck includes a longitudinal open joint, place an expansion joint in the end bent also. In addition, place flashing behind the joint in the end bent. See the INDOT *Standard Drawings*.

### **67-1.07 Concrete**

1. Integral End Bent. All concrete shall be Class C.
2. Non-Integral End Bent. All concrete shall be Class A, unless the concrete is detailed to be poured with the deck. If so, the class of concrete shall be the same as that of the deck.
3. Wingwalls. For a non-integral end bent, the concrete shall be Class A unless otherwise designated. For an integral end bent, the concrete shall be Class C.

## **67-2.0 CANTILEVER ABUTMENT AND WINGWALLS**

See Section 59-2.0 for more information on the selection and design of abutments.

### **67-2.01 Usage**

For soil conditions or bridge geometric dimensions not suitable for spill-through end bents or mechanically stabilized earth abutments, standard abutments with wingwalls of the cantilever type shall be used. These cantilever structural units shall be founded on spread footings, drilled shafts, or driven pile footings with a minimum of two rows of piles. The front row of piles may be battered a maximum of 1:4 (H:V) to provide additional horizontal resistance.

### **67-2.02 Loads**

An abutment stem shall be designed for the imposed gravitational loads, weight of stem, and horizontal loads. The static earth pressure shall be determined in accordance with Article 3.11 of the *LRFD Bridge Design Specifications*. No passive earth pressure shall be assumed to be generated by the prism of earth in front of the wall.

### **67-2.03 General Design and Detailing Criteria**

The following applies to the design and detailing of abutments and wingwalls.

1. Integral Abutment. An integral abutment shall be analyzed as part of a rigid frame for a bridge of 15 m or longer. For a shorter bridge, the abutment may be analyzed as if it is pinned at the top.
2. Expansion Joints. Vertical expansion joints should be considered for an abutment width exceeding 30 m.
3. Abutment/Wingwall. The junction of the abutment wall and wingwall is a critical design element, requiring the considerations as follows:
  - a. If the abutment wall and wingwall are designed using active earth pressure, the two elements shall be separated by a 13-mm filled expansion joint to permit the expected deformations. If the abutment is designed using at-rest earth pressure, no expansion joint between the wingwall and abutment wall is required.
  - b. If the wingwall is tied to the abutment wall (no joint), all horizontal steel reinforcement must be developed into both elements such that full moment resistance can be obtained.
4. Stem Batter. A vertical stem, with no batter, should be used. Where a batter is used, it should be between 1:10 to 1:15 (H:V).
5. Concrete. For the abutment and wingwalls, use Class A concrete for all components above the footings. Use Class B concrete in the footings.
6. Keyways. Keyways shall be used for in vertical expansion and construction joints. See the INDOT *Standard Drawings* for details.
7. Backfill. The abutment and wingwalls shall be backfilled with structure backfill. The neat line limits and the estimated quantity of structure backfill shall be shown on the Layout Sheet of the plans.
8. Reinforcing Steel. If an expansion joint is located directly over the abutment cap, all reinforcement in the abutment wall shall be epoxy coated.
9. Toe. The fill on the toe of footing should be ignored.
10. Soil Weight. Only the weight of the soil vertically above the heel of the footing shall be included in the overturning stability analysis and the structural design of the footing.
11. Minimum Thickness. The minimum thickness of footings shall be 450 mm.

12. Piles. Footings on piles should be analyzed ignoring the structural contribution of the concrete below the top of piles.
13. Loads. Pile loads shall be computed by using the formula as follows:

$$B = \frac{V}{N} + \frac{(V)(x)(e)}{I}$$

Where:

- B = load on piles (kN)
- V = vertical component of resultant (kN)
- N = number of piles
- e = distance from center of gravity of pile group to point where resultant strikes the top of pile elevation (m)
- x = distance from center of gravity of pile group to the pile under consideration (m)
- I = moment of inertia of pile group about center of gravity of group with each pile taken as a unit (m<sup>2</sup>)

14. Figures. Figures 67-2A through 67-2C illustrate the preferred method for determining the geometrics for a flared wingwall.

#### **67-2.04 Drainage**

Positive drainage shall be provided behind each abutment and/or wingwalls. See the INDOT *Standard Drawings* for a weephole detail. Drains shall be located in abutments and wingwalls as follows:

1. Abutment (wingwall of 4.5 m or shorter). Omit drains in wingwalls, and space them at 3.5 m maximum in the abutment.
2. Abutment (wingwalls longer than 4.5 m). Use a 3.5-m drain maximum spacing with a 3.5-m maximum distance from the ends of the wingwalls.
3. Location of Drain Outlet. Locate the outlet 300 mm above the low water elevation or the proposed ground line elevation.

#### **67-2.05 Construction Joints**

Use construction joint type A for all horizontal construction joints in both the abutment and wingwalls. See the INDOT *Standard Drawings*. Vertical construction joints should be placed as follows:

1. In an Abutment. Preferably at 10.0 m center to center, with a maximum of 12 m.
2. In Wingwall of 6 m or Longer. In the wingwall section at 6.0 m center to center and cut one batter face only.
3. In Wingwall Shorter than 6 m. In the abutment section so that the combined length of wingwall and abutment between joints is approximately 6.0 m.
4. In Either the Wingwall or the Abutment. Not less than 450 mm from the intersection of batter faces at the top of the footing.

Joints should not be placed under bridge bearing areas.

The horizontal reinforcing should continue through the construction joint. Vertical bars should be placed at a minimum of 75 mm from the centerline of joint.

### **67-2.06 Details**

Figure 67-2D provides typical abutment details.

## **67-3.0 INTERIOR SUPPORTS**

### **67-3.01 Types**

Three basic types of interior supports are used, which are discussed in the following sections. Also, see Sections 59-2.0 and 66-3.04 for more information.

#### **67-3.01(01) Extended Pile or Drilled Shaft Bent**

Under certain conditions, the economy of a substructure can be enhanced by extending a deep foundation, such as a single row of driven piles or drilled shafts above ground level to the superstructure. Extended pile bents may be of the integral type or the non-integral type. See Figure 67-3A.

### **67-3.01(02) Stem-Type Piers**

The types of stem piers are as follows:

1. Single-Wall. This is a relatively thin wall set on a single row of piles, a spread footing, or a pile cap with multiple rows of piles. The single-wall is most suitable if its structural height is less than 6 m, or the superstructure length is less than 8 m parallel to the pier centerline. See Figure 67-3B for a wall pier on a single row of piles.
2. Hammerhead. For a larger structural height or pier width, a hammerhead pier, either with rectangular or rounded stem, is often more suitable. Figure 67-3C illustrates the typical design of a hammerhead pier.

### **67-3.01(03) Frame Bents**

Concrete frame bents may be used to support a variety of superstructures. The columns of the bent may be either circular or rectangular in cross section. The columns may be directly supported by the footing or by the partial height wall. Figures 67-3D and 67-3E illustrate the typical designs of frame bents. If the columns rest directly on the footing, the footing shall be designed as a two-way slab. Construction joints may be required in the cap if the concrete shrinkage moments introduced into the columns becomes excessive.

### **67-3.02 Usage**

The selection of the interior support type should be based on the feature passing beneath the bridge, as follows:

1. Major Water Crossing. A hammerhead, wall, or single round column-type pier supported by a deep foundation or a spread footing on rock is preferred for a major water crossing. Multiple round columns may be used, but they may require a solid wall between columns to avoid the collection of debris. This decision must be coordinated with the Design Division's Hydraulics Unit. A single wall pier may be a more suitable alternative.
2. Meandering River. For a meandering river or stream, or where the high flow is at a different skew than the low flow, the most desirable pier type is normally a single, circular pier column.
3. Highway Grade Separation. Thin walls or frame bents with multiple columns should be used. The aesthetics of the pier should be considered. Special surface treatments using form liners or other means should be investigated, especially for a wall pier.

### **67-3.03 General Design Considerations**

The following design criteria apply to interior supports, where applicable.

1. Pier in Waterway. A stem-type pier shall have a solid wall to an elevation of 300 mm above the  $Q_{100}$  high-water level. Depending on aesthetics and economics, the remainder of the wall may be either solid or multiple columns. The dimensions of the wall may be reduced by providing cantilevers to form a hammerhead pier.
2. Railroad Crossing. The design shall meet AREMA requirements if the pier is within 7.5 m of the present track or future track centerlines. If the pier is located within a distance of 15 m from the centerline of a railroad track, it should be designed for the collision force, if applicable, as specified by the *LRFD Bridge Design Specifications*, Article 3.6.5.2.
3. Cap Reinforcement. Multiple layers of negative moment reinforcement are permitted in caps to minimize cap dimensions.
4. Column Reinforcement. Column vertical bars shall extend a minimum of 20 bar diameters into the cap beam and the required development length into the spread footing or pile cap. The area of steel reinforcement provided across the interface between the base of the column or pier stem and the top of footing shall not be less than 0.5% of the gross area of column or stem as specified by the *LRFD Bridge Design Specifications*, Article 5.13.3.8. According to LRFD Article 5.10.11.4.2, the minimum reinforcement ratio, both horizontally and vertically in a pier, shall not be less than 0.0025. The vertical reinforcement ratio shall not be less than the horizontal reinforcement ratio. The reinforcement spacing, either horizontally or vertically, shall not exceed 450 mm.
5. Size. Columns are typically rectangular, square, or round, with a minimum diameter or thickness of 600 mm. Diameter increments shall be in multiples of 150 mm. A solid pier wall shall have a minimum thickness of 600 mm and may be widened at the top to accommodate the bridge seat.
6. Cap Extension. The width of each cap shall project beyond the sides of the columns. The added width of the cap shall be a minimum of 40 mm on the outside the columns. This width will reduce the reinforcement interference between the column and cap. The cap should preferably have cantilevered ends to balance positive and negative moments in the cap.

7. Step Cap. Where one end of the cap is on a considerably different elevation than the other, the difference shall be accommodated by increasing the column heights as shown in Figure 67-3E1. The bottom of the cap shall be sloped at the same rate as the cross slope of the top of the bridge deck. The top of the cap shall be stepped to provide level bearing surfaces.
8. Construction Joints. Use construction joint type A for all horizontal construction joints. See the INDOT *Standard Drawings*. However, at the base of the pier stem, a keyway construction joint shall be used.
9. Reinforcement Clearance. The clearances of the reinforcement should be checked to ensure that there is adequate space for the proper placement of the concrete during construction.
10. Backfill. An interior bent or pier at the base of a slopewall shall be backfilled with structure backfill as shown on the INDOT *Standard Drawings*. For an interior bent or pier adjacent to a railroad, the area should be backfilled with structure backfill to a point 450 mm outside the neat lines of the footing. Show the neat line limits and estimate the quantity on the Layout Sheet for each bent and pier location. Do not provide structure backfill as backfill material around a pier that is located in a stream.
11. Epoxy-Coated Steel Under Expansion Joints. All reinforcing steel in concrete above the footing at an interior pier where an expansion joint is located directly over the cap shall be epoxy coated. This includes the stem, cantilevers, and cap. This applies only to a substructure which supports the ends of two superstructure units with an expansion joint located directly over the cap.
12. Concrete. For an interior support, use Class A concrete above the footings, and use Class B concrete in footings.
13. Steel Splices. If a pier stem is less than 3 m in height, do not splice the steel extending out of the footing. For small columns with a high percentage of vertical steel and for columns in Seismic Zone 2, mechanical connectors should be used for splicing the vertical steel.
14. Compression Steel. Compression steel tends to buckle once the concrete cover is gone or where the concrete around the steel is weakened by compression. The criteria in the *LRFD Specifications*, Articles 5.7.4.6, 5.10.6, or 5.10.11, for ties and spirals should be used. See Figure 67-3F for suggested hammerhead and wall type pier reinforcing in columns with no plastic hinging capability. Ties may be #10 bars for longitudinal bars up to size #32.

### **67-3.04 Specific Design Criteria**

This Section describes design criteria which applies to each specific type of interior support.

#### **67-3.04(01) Extended Pile Bent**

The following applies to the design of an extended pile bent.

1. Limitations. This type of support has very little resistance to longitudinal forces and should not be used unless these forces are resisted by other substructure units such as integral end bents or abutments. This support should also not be used if the stream carries large debris, heavy ice flow, or large vessels. If steel H-piles are used for support, they shall be encased in concrete. The concrete encasement shall be extended to 600 mm below the flow line elevation. Encasement details are provided on the INDOT *Standard Drawings*. Scour should be considered in establishing design pile lengths and for the structural design of the pile.
2. Cap Beam. Extended piles always need a cap beam for structural soundness, which may be an integral part of the superstructure. Extended drilled shafts could be arranged to support, for example, widely spaced beams without the presence of a cap beam if sufficient space is provided at the top for mandatory jacking operations.
3. Loads. Girders may be fixed or semi-fixed at an extended pile bent. Because the piles are relatively flexible compared to the end bents or abutments, the force effects induced in the piles by lateral displacement is small. Where practical, one pile should be placed beneath each girder. The vertical load carried by the piles shall be the girder reaction and the appropriate portion of the pile cap dead load. Assuming the bent acts as a rigid frame in a direction parallel to the bent, force effects due to lateral displacement and lateral loads may be uniformly distributed among the extended piles.
4. Cap Design. The minimum reinforcement for the concrete bent cap shall be #16 bars at 300-mm spacing on all faces and shall satisfy Article 5.7.3.3 of the *LRFD Bridge Design Specifications*. The cap shall be designed as a continuous beam.

#### **67-3.04(02) Hammerhead Pier**

The following applies to the design of a hammerhead pier.

1. Cofferdam. If a cofferdam is anticipated to be required, the hammerhead portion of the pier must be above the average low-water level of the stream.
2. Bottom Elevation. The bottom of the hammerhead portion of the pier should preferably be a minimum of 2 m above the finished ground line at a stream crossing to help prevent debris accumulation.
3. Effective Length Factor. The commentary table in Article 4.6.2.5 of the *LRFD Bridge Design Specifications* presents criteria for the effective length factor, K. For beams on rockers and slide bearings, use  $K = 2.1$ . For an expansion pier with beams on a single row of neoprene pads, use  $K = 1.5$ . For semi-fixed (prestressed concrete beams) on a fixed pier, use  $K = 1.2$ . Use  $K = 1.0$  for the strong or transverse direction.
4. Pier Walls. These should be designed as columns for bi-axial bending.

### **67-3.04(03) Frame Bent**

The following applies to the design of a frame bent.

1. Column Fixity. The columns founded on a spread or multiple pile footing shall be assumed to be fixed at the bottom.
2. Cantilevered Cap. The moments used for the cap design shall be calculated at the face of the support for a square or rectangular column or at the theoretical face of a circular column.
3. Effective Length Factor. Use the same K factors as described for hammerhead piers in Section 67-3.04(02) in the weak (longitudinal) direction. Use  $K = 1.0$  for the strong (transverse) direction.
4. Structural Design. If the number of columns is kept to a minimum, and the components are reasonably small, frame analysis is both appropriate and safe for frame bents.

### **67-3.05 Compression**

Reinforced concrete piers, pier columns, and piles are referred to as compressive members although their design is normally controlled by flexure. Tall, slender columns or pier shafts are relatively rare due to topography. The use of the moment magnification approach in the *LRFD Specifications* is most often warranted. For exceptionally tall or slender columns or shafts, a refined analysis, as outlined in Article 5.7.4.1 of the *LRFD Specifications*, should be performed.

For a bridge in Seismic Zone 1 and 2, a reduced effective area may be used when the cross section is larger than that required to resist the applied loading. The minimum percentage of total longitudinal reinforcement of the reduced effective area is to be the greater of 1% or the value obtained from Equation 5.7.4.2-3 in the *LRFD Specifications*. Both the reduced effective area and the gross area must be capable of resisting all applicable load combinations shown in Table 3.4.1-1 of the *LRFD Specifications*.

## **67-4.0 BEARINGS**

### **67-4.01 General**

Bearings ensure the functionality of a bridge by allowing translation and rotation to occur while supporting the vertical loads. However, the designer should first consider the use of integral abutments and possibly integral piers prior to deciding upon the use of bearings to support the structure. The following will apply.

1. Movements. Movement should be considered. Movements include both translations and rotations. The sources of movement include bridge skew and horizontal curvature effects, initial camber or curvature, construction loads, misalignment or construction tolerances, settlement of supports, thermal effects, creep, shrinkage, and traffic loading. Bearing pads on a skewed structure should be oriented parallel to the principal rotation axis. Where insufficient seat width exists, the bearing pads may be oriented normal to the support.
2. Effect of Bridge Skew and Horizontal Curvature. A skewed bridge moves both longitudinally and transversely. The transverse movement becomes significant on a bridge with a skew angle greater than 20 degrees that has bearings not oriented parallel to the movement of the structure.

A curved bridge moves both radially and tangentially. These complex movements are predominant in a curved bridge with small radii and with expansion lengths that are longer than 60 m.

3. Effect of Camber and Construction Procedures. The initial camber of bridge girders and out-of-level support surfaces induces bearing rotation. Initial camber may cause a larger initial rotation on the bearing, but this rotation may grow smaller as the construction of the bridge progresses. Rotation due to camber and the initial construction tolerances is sometimes the largest component of the total bearing rotation. Both the initial rotation and its short duration should be considered. If the bearing is installed level at an intermediate state of construction, deflections and rotations due to the weight of the deck

slab and construction equipment must be added to the effects of live load and temperature. Construction loads and movements due to tolerances should be included. The direction of loads, movements, and rotations must also be considered, because it is inappropriate to simply add the absolute magnitudes of these design requirements. Rational design requires consideration of the worst possible combination of conditions without designing for unrealistic or impossible combinations or conditions. It may often be economical to install the bearing with an initial offset, or to adjust the position of the bearing after construction has started, to minimize the adverse effect of these temporary initial conditions. Combinations of load and movement that are not possible should not be considered.

4. Thermal Effects. Thermal translations,  $\Delta o$ , are estimated as follows:

$$\Delta o = \alpha L \Delta T$$

where  $L$  is the expansion length,  $\alpha$  is the coefficient of thermal expansion of  $10.8 \times 10^{-6} / ^\circ\text{C}$  for normal density concrete or  $11.7 \times 10^{-6} / ^\circ\text{C}$  for steel, and  $\Delta T$  is the change in the average bridge temperature from the installation temperature. A change in the average bridge temperature causes a thermal translation. A change in the temperature gradient induces bending and deflections. The design temperature changes are specified by the *AASHTO LRFD Specifications*. Maximum and minimum bridge temperatures are defined depending upon whether the location is viewed as a cold or moderate climate. Indiana is considered a cold climate. See LRFD Table 3.12.2.1-1 for temperature range values. The designer should assume an installation temperature of  $15^\circ\text{C}$ . The change in average bridge temperature,  $\Delta T$ , between the installation temperature and the design extreme temperatures is used to compute the positive and negative movements. A given temperature change causes thermal movement in all directions. This means that a short, wide bridge may experience greater transverse movement than longitudinal movement.

5. Loads and Restraint. Restraint forces occur when any part of a movement is prevented. Forces due to direct loads include the dead load of the bridge and loads due to traffic, earthquakes, water, and wind. Temporary loads due to construction equipment and staging also occur. The majority of the direct design loads are reactions of the bridge superstructure on the bearing, so they can be estimated from the structural analysis. The applicable AASHTO load combinations must be considered.
6. Serviceability, Maintenance, and Protection Requirements. Bearings under deck joints collect large amounts of dirt and moisture which promotes problems of corrosion and deterioration. As a result, these bearings should be designed and installed to have the maximum possible protection against the environment and to allow easy access for inspection.

The service demands on bridge bearings are very severe and result in a service life that is typically shorter than that of other bridge elements. Therefore, allowances for bearing replacement should be part of the design process. Lifting locations should be provided to facilitate removal and re-installation of bearings without damaging the structure. No additional hardware is usually necessary for this purpose. The primary requirements are to allow space suitable for lifting jacks during the original design and to employ details that permit quick removal and replacement of the bearing.

7. Clear Distance. The minimum clear distance between the bottom shoe of a steel bearing and the edge of the bearing seat or cap shall be 75 mm. For an elastomeric pad resting directly on the concrete bridge seat, the minimum edge distance shall be 150 mm under a deck expansion joint and 75 mm (100 mm desirable) for all other locations. Seismic support lengths should also be checked.
8. Bearing Selection. Bearing selection is influenced by many factors such as loads, geometry, maintenance, available clearance, displacement, rotation, deflection, availability, policy, designer preference, construction tolerances, and cost.

Vertical displacements are prevented, rotations are allowed to occur as freely as possible, and horizontal displacements may be either accommodated or prevented. The loads should be distributed among the bearings in accordance with the superstructure analysis.

Unless conditions dictate otherwise, conventional steel reinforced elastomeric bearings should be used for a girder bridge. Where the practical limits of an elastomeric bearing pad are exceeded, the designer should consider using flat polytetrafluorethylene (PTFE) slider plates in conjunction with a steel-reinforced elastomeric bearing. See Figure 67-4A for a general summary of expansion bearing capabilities. The values shown in the table are for guidance only.

The final step in the selection process consists of completing a design of the bearing in accordance with the *LRFD Specifications*. The resulting design will provide the geometry and other pertinent specifications for the bearing.

For a structure widening, bearing types should not be mismatched. Yielding type bearings, such as elastomeric, should not be used in conjunction with steel rockers or other non-yielding type bearings.

A steel beam bridge without integral end bents must have at least one fixed bearing line. Due to the interior diaphragm keyway, semi-fixed interior supports are allowed for a precast concrete bridge. If integral end bents meeting the empirical design limits are used, interior fixed bearings are not required.

9. Anchor Plates/Anchor Bolts. Anchor plates should only be used to attach the bottom steel shoe of an expansion bearing to the concrete beam seat. Anchor bolts shall be used to connect fixed steel bearings to the concrete beam seat.

#### **67-4.02 Fixed Steel Bearings**

The top shoe of a steel bearing shall be at least as wide as the beam flange, but not more than 50 mm wider. The maximum reaction is given for each shoe type in the INDOT *Standard Drawings*. An independent design is required if the design reaction is greater than the maximum reaction shown or if the beam or girder flange width does not meet the requirements of the *Standard Drawings*.

If the flexibility of tall, slender piers is sufficient to absorb the horizontal movement at the bearings due to temperature change without developing undue force in the superstructure, bearings, or pier, two or more piers may be fixed to distribute the longitudinal force among the piers.

#### **67-4.03 Elastomeric Bearing Pads and Steel Reinforced Elastomeric Bearings**

Elastomers are used in both elastomeric bearing pads and steel reinforced elastomeric bearings. The behavior of both pads and bearings is influenced by the shape factor ( $S$ ), as follows:

$$S = \frac{\text{Plan Area}}{\text{Area of Perimeter Free to Bulge}}$$

Elastomeric bearing pads and steel reinforced elastomeric bearings have fundamentally different behaviors and, therefore, they are discussed separately. It is usually desirable to orient elastomeric pads and bearings so that the long side is parallel to the principal axis of rotation, because this facilitates the accommodation of rotation.

Holes should not be placed in an elastomeric bearing pad due to increased stress concentrations around the hole. These increased stresses can cause tearing of the elastomer during extreme events, such as an earthquake. If holes are placed in a steel reinforced bearing, the steel reinforcement thickness should be increased in accordance with LRFD Article 14.7.5.3.7.

Elastomeric bearing pads and steel reinforced elastomeric bearings have many desirable attributes. They are usually a low-cost option, and they require minimal maintenance. Further, these components are relatively forgiving if subjected to loads, movements, or rotations that are slightly larger than those considered in their design. This is not to encourage underdesign of elastomeric pads and bearings, but it is to note that extreme events, which have a low probability

of occurrence, will have far less serious consequences with these elastomeric components than with other bearing systems.

#### **67-4.03(01) Elastomer**

Both natural rubber and neoprene are used in the construction of bridge bearings. The differences between the two are usually not very significant. Neoprene has greater resistance than natural rubber to ozone and a wide range of chemicals, so it is more suitable for some harsh chemical environments. However, natural rubber generally stiffens less than neoprene at low temperatures.

All elastomers are visco-elastic, nonlinear materials, and therefore, their properties vary with strain level, rate of loading, and temperature. Bearing manufacturers evaluate the materials on the basis of Shore A Durometer hardness, but this parameter is not a good indicator of the shear modulus,  $G$ . A Shore A Durometer hardness of  $55 \pm 5$  should be used. This leads to shear modulus values in the range of 0.78 to 1.14 MPa at 23°C. The least favorable value should be used for design. The shear stiffness of the bearing is its most important property because it affects the forces transmitted between the superstructure and substructure.

Elastomers are flexible under shear and uniaxial deformation, but they are very stiff against volume changes. This feature makes possible the design of a bearing that is stiff in compression but flexible in shear.

Elastomers stiffen at low temperatures. The low-temperature stiffening effect is very sensitive to elastomer compound and the increase in shear resistance can be controlled by selection of an elastomer compound that is appropriate for the climatic conditions. The minimum low-temperature elastomer shall be Grade 3, unless otherwise specified. The elastomer grade should be shown in the contract documents.

#### **67-4.03(02) Steel Reinforced Elastomeric Bearing Pads**

Steel reinforced elastomeric bearings are often categorized with elastomeric bearing pads, but the steel reinforcement makes their behavior quite different. Steel reinforced elastomeric bearings have uniformly spaced layers of steel and elastomer. The bearing accommodates translation and rotation by deformation of the elastomer. The elastomer is flexible under shear stress but stiff against volumetric changes. Under uniaxial compression, the flexible elastomer would shorten significantly and sustain large increases in its plan dimension, but the stiff steel layers restrain this lateral expansion. This restraint induces a bulging pattern as shown in Figure 67-4B and provides a large increase in stiffness under compressive load. This permits a steel reinforced

elastomeric bearing to support relatively large compressive loads while accommodating large translations and rotations.

The design of a steel reinforced elastomeric bearing pad requires an appropriate balance of compressive, shear, and rotational stiffnesses. The shape factor affects the compressive and rotation stiffness, but it has no impact on the translational stiffness or deformation capacity.

A bearing pad must be designed to control the stress in the steel reinforcement and the strain in the elastomer. This is done by controlling the elastomer layer thickness and the shape factor of the bearing. Fatigue, stability, delamination, yield, and rupture of the steel reinforcement, stiffness of the elastomer, and geometric constraints must be satisfied.

Large rotations and translations require thicker bearings. Translations and rotations may occur about the longitudinal or transverse axis of a steel reinforced elastomeric bearing.

A steel reinforced elastomeric bearing becomes large if it is designed for a load greater than about 3000 kN. Uniform heating and curing during vulcanization of such a large mass of elastomer becomes difficult, as elastomers are poor heat conductors. Manufacturing constraints thus impose a practical upper limit on the size of most steel reinforced elastomeric bearings. If the design load exceeds 3000 kN, the designer should check with the manufacturer for availability.

#### **67-4.03(03) Elastomeric Bearing Pads**

Elastomeric bearing pads include plain elastomeric pads (PEP), cotton-duck reinforced pads (CDP), and layered fiberglass reinforced bearing pads (FGP). The designer must prepare a unique special provision if any of these pads are to be used. There is considerable variation between pad types. Elastomeric bearing pads can support modest gravity loads, but they can only accommodate limited rotation or translation. Hence, they are best suited for a bridge with small expansion lengths or a specialty situation.

A plain elastomeric pad relies on friction at its top and bottom surfaces to restrain bulging due to the Poisson effect. Friction is unreliable, and local slip results in a larger elastomer strain than that which occurs in a reinforced elastomeric pad or bearing. The increased elastomer strain limits the load capacity of the PEP. The PEP must be relatively thin if it will carry the maximum allowable compressive load. A maximum friction coefficient of 0.20 should be used for the design of an elastomeric pad that is in contact with clean concrete or steel surfaces. If the shear force is greater than 0.20 of the simultaneously occurring compressive force, the bearing should be secured against horizontal movement. If the designer is checking the maximum seismic forces that can be transferred to the substructure through the pad, a friction coefficient of 0.40 should be used.

A cotton-duck reinforced pad has very thin elastomer layers (less than 0.4 mm). It is stiff and strong in compression so it has much larger compressive load capacities than a PEP, but it has very little rotational or translational capacity. A CDP is sometimes used with a PTFE slider to accommodate horizontal translation.

The behavior of an elastomeric pad reinforced with discrete layers of fiberglass (FGP) is closer to that of a steel reinforced elastomeric bearing than to that of other elastomeric bearing pads. The fiberglass, however, is weaker, more flexible, and bonds less well to the elastomer than does the steel reinforcement. Sudden failure occurs if the reinforcement ruptures. These factors limit the compressive load capacity of the fiberglass reinforced bearing pad. An FGP pad accommodates larger gravity loads than a PEP of identical geometry, but its load capacity may be smaller than that achieved with a CDP. An FGP can accommodate modest translations and rotations.

The use of an FGP or PEP elastomeric pad is restricted to lighter bearing loads, on the order of 700 kN or less. A CDP may support somewhat larger loads than an FGP or PEP, on the order of 1200 kN or less, but has no significant translation or rotational capabilities. Translations of 20 mm and rotations of one degree or less are possible with an FGP or a PEP bearing.

Due to the limited use of PEP, FGP, and CDP bearing pads, no design example will be given in this *Manual*. See the *LRFD Specifications* Article 14.7.6 for design requirements.

#### **67-4.04 Design of a Steel Reinforced Elastomeric Bearing Pad**

A steel reinforced elastomeric bearing may be designed using either of two methods commonly referred to as Method A and Method B. The Method A procedure found in *LRFD Specifications* Article 14.7.6 shall be used for a conventional elastomeric bearing. The Method B procedure found in *LRFD Specifications* Article 14.7.5 may be used for a high-capacity bearing.

The Method B design procedure allows significantly higher average compressive stresses. These higher allowable stress levels are justified by an additional acceptance test, specifically a long-duration compression test. A unique special provision must be developed if a high-capacity elastomeric bearing is to be used. A high-capacity elastomeric bearing of *LRFD* Method B Design should be used only where very tight geometric constraints, extremely high loads, or special conditions or circumstances require the use of higher grade material. The use of the Method B design will require the approval of the Design Division Chief.

Design criteria for both methods are based on satisfying fatigue, stability, delamination, steel reinforcement yield/rupture, and elastomer stiffness requirements. The design of a steel reinforced elastomeric bearing requires an appropriate balance of compressive, shear, and

rotational stiffnesses. The shape factor, as defined by the steel shim spacing, significantly affects the compressive and rotational stiffness of the bearing. However, it has no impact on the translational stiffness of the bearing or its translational deformation capacity. Section 67-4.05 presents a design example for a steel reinforced elastomeric bearing pad using the Method A design procedure.

The minimum elastomeric bearing length or width shall be 150 mm. The durometer hardness shall be  $55 \pm 5$ . For an overall bearing thickness of less than 88 mm, a minimum of 3 mm of side clearance shall be provided beyond the edges of the steel shims. For an overall thickness of 88 mm or greater, a minimum of 6 mm of side clearance shall be provided. The top and bottom cover layers shall not be more than 70 percent of the thickness of the interior layers.

In determining bearing pad thickness, it should be assumed that slippage will not occur. The total elastomer thickness shall not be less than twice the maximum longitudinal or transverse deflection. The bearing should be checked against horizontal crawling in accordance with LRFD Article 14.7.6.4.

AASHTO places Indiana in a cold climate. A setting temperature of 15°C shall be used for the installation of the bearing. The formulas for determining the total elastomer thickness are shown below.

1. For precast concrete girder spans, an allowance must be made for half of the shrinkage. The change in length ( $\Delta L_c$ ) shall be based on a 33°C temperature differential.
2. Minimum total elastomer thickness for a precast girder is  $2(\Delta L_c + \Delta \frac{1}{2} \text{ Shrinkage})$ .
3. For steel-girder spans, the change in length ( $\Delta L_s$ ) shall be based on a 50°C temperature differential. No allowance is needed for shrinkage.
4. Minimum total elastomer thickness for a steel girder is  $2\Delta L_s$ .
5. For cast-in-place and post-tensioned concrete spans, the full shrinkage, elastic shortening, and creep shall be considered.

#### **67-4.04(01) Determination of the Minimum Total Elastomer Thickness for a Precast Concrete Beam**

Example: Find the change in length due to thermal effects ( $\Delta L_c$ ) and the shrinkage (SR) to determine the minimum total elastomer thickness ( $h_{rt}$ ) for a steel reinforced elastomeric bearing pad. The precast, prestressed member is 30 m long with the same expansion length (L), has a volume-to-surface area ratio of 100 mm, and is evaluated at a 10,000-day drying time.

Conservatively, assume a stream-cured concrete as follows:

$$h_{rt} \geq 2 (\Delta L_c + \Delta \frac{1}{2} SR)$$

$\Delta L_c$  can be calculated by using the standard concrete thermal expansion coefficient  $\alpha = 10.8 \times 10^{-6}/^\circ\text{C}$ , as follows:

$$\Delta L_c = \alpha L \Delta T = (0.0000108)(30\,000\text{ mm})(33^\circ\text{C}) = 10.7\text{ mm}$$

The total shrinkage,  $SR = L_{\text{beam}} \times \gamma_{\text{sh}}$ .

First, find the strain attributed to shrinkage for steam-cured concrete,  $\gamma_{\text{sh}}$ , as follows:

$$\epsilon_{\text{sh}} = -k_s k_h \left( t / (55.0 + t) \right) (0.56 \times 10^{-3}) \quad (\text{LRFD Equation 5.4.2.3.3-2})$$

where  $k_s$  is the size factor,  $k_h$  is the humidity factor, and  $t$  is the drying time (days).

Since  $\frac{\text{Volume}}{\text{Surface Area}} = 100\text{ mm}$ ,  $k_s = 0.75$  at 10 000 days (LRFD Figure 5.4.2.3.3-2)

Since Indiana has an Annual Average Ambient Relative Humidity of approximately 73% (from LRFD Figure 5.4.2.3.3-1), interpolate  $k_h = 0.96$  from LRFD Table 5.4.2.3.3-1.

Therefore:

$$\epsilon_{\text{sh}} = (-0.75)(0.96)(10\,000/(55 + 10\,000)) (0.56 \times 10^{-3}) = -4.010 \times 10^{-4}$$

and

$$SR = (-4.010 \times 10^{-4})(30\text{ m}) = -0.0120\text{ m or } -12\text{ mm}$$

The minimum total elastomer thickness is then determined by using this 12-mm *additive* to beam contraction, as follows:

$$h_{rt} \geq 2 (\Delta L_c + \Delta \frac{1}{2} SR) = 2(10.7\text{ mm} + 12\text{ mm}/2) = 33.4\text{ mm}$$

Use:  $h_{rt} \geq 34\text{ mm}$

#### **67-4.05 Steel Reinforced Elastomeric Bearing Pad (Design Example)**

The following example is based on LRFD, Article 14.7.6, Design Method A.

Given:

A 3-span bridge with integral end bents and W920x201 Grade 345 steel beams. Bridge spans are 19 m, 23 m, and 19 m on a 1% grade. The bottom flange width of the beam is 304 mm. No holes are drilled through the pads. Use a temperature range of -35° C to 50° C for steel or -18° C to 27° C for concrete (AASHTO Table 3.12.2.1-1).

Problem:

Use Method A. Load modifiers,  $\eta_i$ , for ductility, redundancy, and operational importance are assumed to be equal to 1.0 for this Example. The resistance factor for bearings,  $\phi$ , shall be taken as 1.0 (Article 14.6.1).

Solution:

1. Thermal expansion coefficient  $\alpha = 11.7 \times 10^{-6}/^{\circ}\text{C}$ ; since this is a steel beam. For normal density concrete,  $\alpha = 10.8 \times 10^{-6}/^{\circ}\text{C}$  would be used.

2. The Standard Shore A Durometer hardness of 55 should be used. Therefore, the shear modulus range is as follows:

$$G_{\min} = 0.78 \text{ MPa} \quad (\text{AASHTO Table 14.7.5.2-1})$$

$$G_{\max} = 1.14 \text{ MPa} \quad (\text{AASHTO Table 14.7.5.2-1})$$

3. Unfactored load reactions from the structural analysis are as follows:

$$\text{Non-composite dead loads (beam + deck + forms)} \quad P_{\text{nd}} = 376.32 \text{ kN}$$

$$\text{Composite dead loads (FWS not included)} \quad P_{\text{cd}} = 31.39 \text{ kN}$$

$$\text{Future wearing surface load} \quad P_{\text{fws}} = 66.91 \text{ kN}$$

$$\text{Maximum live load*} \quad P_{\text{live}} = 213.87 \text{ kN}$$

$$\text{Maximum live load uplift} \quad P_{\text{uplift}} = 14.19 \text{ kN}$$

*\*Impact is not included in Service I per AASHTO 14.4.1.*

4. With a standard setting temperature of 15° C and minimum -35° C based on a cold climate,

$$\Delta T = 50^{\circ} \text{ C} \quad (\text{AASHTO Table 3.12.2.1-1})$$

5. Pad Properties: Try a width 25 mm to 50 mm wider than bottom flange. Provide top shoe. (For a concrete beam, the pad width shall be at least 50 mm less than bottom flange width) (no top shoe).

$$\text{Choose } W = 350 \text{ mm.}$$

The required length can be estimated using AASHTO Equation 14.7.6.3.2-4. Assume that allowable compressive stress of 7 MPa will govern as follows:

$$L_{\text{dreq}} = \frac{(P_{\text{nd}} + P_{\text{cd}} + P_{\text{fws}} + P_{\text{live}})}{W(7 \text{ MPa})}$$

$$L_{\text{dreq}} = \frac{(376.32 \text{ kN} + 31.39 \text{ kN} + 66.91 \text{ kN} + 213.87 \text{ kN})1000}{[350 \text{ mm} (7 \text{ MPa})]}$$

$$L_{\text{dreq}} = 281 \text{ mm}$$

Choosing  $L = 325 \text{ mm}$ ,

Is  $L > L_{\text{dreq}}$ ? Check 1: OK

And  $A_{\text{pad}} = LW = (325 \text{ mm})(350 \text{ mm}) = 113\,750 \text{ mm}^2$

Investigate a trial pad cross-section (*LRFD* 14.7.6.1). Note that the top and bottom elastomer layers must each be less than 70% of an interior layer thickness.

- |    |  |                                     |
|----|--|-------------------------------------|
| a. | Thickness of interior elastomer layer:                     | $h_{\text{ri}} = 10 \text{ mm}$     |
| b. | Thickness of exterior elastomer layer:                     | $h_{\text{re}} = 5 \text{ mm}$      |
| c. | Thickness of steel shim:                                   | $h_{\text{s}} = 2.5 \text{ mm}$     |
| d. | Total number of shims:                                     | $n_{\text{shims}} = 7$              |
| e. | The number of interior elastomer layers in this bearing:   | $n_{\text{layers}} = 6$             |
| f. | Since we consider the thickest elastomer layer for design: | $h_{\text{rmax}} = h_{\text{ri}}$   |
| g. | The total thickness of elastomer in this bearing:          | $h_{\text{rt}} = 70.00 \text{ mm}$  |
| h. | Total pad height:  | $h_{\text{pad}} = 87.50 \text{ mm}$ |

See Figure 67-4D.

6. Rotation (LL without Impact): From rotational analyses, LL + DL rotation is as follows:

$$\theta_x = 0.002 \text{ rad}$$

$$\text{Slope of the 1\% gradeline} = 0.010 \text{ rad}$$

If the grade is greater than approximately 1%, a tapered shim should be considered.

*LRFD* 14.4.2 designates a minimum allowance for uncertainties of 0.005 rad. The total unfactored service rotation about the transverse axis is as follows:

$$\theta_{\text{sx}} = \theta_x + \text{grade} + 0.005 \text{ rad} = 0.002 \text{ rad} + 0.010 \text{ rad} + 0.005 \text{ rad} = 0.017 \text{ rad}$$

Since no torsional effects are considered,  $\theta_{\text{sz}} = 0.005 \text{ rad}$ .

7. Expansion Length: The effective expansion length of the beam at an interior support associated with this pad is as follows:

$$L_e = \frac{23\,000\text{ mm}}{2} = 11\,500\text{ mm}$$

Expansion or contraction due to shrinkage and creep is as follows:

$$\Delta_{sc} = 0\text{ mm}$$

### Design Checks:

1. Shape Factor (*LRFD* Eq. 14.7.5.1-1):

$$S = \frac{LW}{2h_{ri}(L+W)} = \frac{(325\text{ mm})(350\text{ mm})}{2(10\text{ mm})(325\text{ mm} + 350\text{ mm})} = 8.43$$

2. Compressive Stress (*LRFD* Eq. 14.7.6.3.2-4) is as follows:

- Total allowable stress  $\sigma_s < 1.0GS$  and  $\sigma_s < 7\text{ MPa}$
- Allowable stress:  $G_{min}S = (0.78\text{ MPa}) 8.43 = 6.57\text{ MPa} < 7\text{ MPa}$ ?
- For this situation, the maximum allowable stress is  $\sigma_{allow} = 6.57\text{ MPa}$
- The compressive stress at service limit state:

$$\sigma_s = \frac{(P_{nd} + P_{cd} + P_{fws} + P_{live})}{LW} = \frac{(376.32\text{ kN} + 31.39\text{ kN} + 66.91\text{ kN} + 213.87\text{ kN})1000}{(325\text{ mm})(350\text{ mm})}$$

$$\sigma_s = 605\text{ MPa}$$

$$\text{Is } \sigma_s < \sigma_{allow}?$$

Check 2: OK

3. Find compressive deflection using *LRFD* 14.7.5.3.3 and 14.7.6.3.3.  
(These calculations are unnecessary unless deck joints and seals are used.)

Service Load Deflection:

$$\text{With } \frac{(P_{nd} + P_{cd} + P_{fws})}{LW} = \left[ \frac{376.32\text{ kN} + 31.39\text{ kN} + 66.91\text{ kN}}{(325\text{ mm})(350\text{ mm})} \right] 1000 = 4.17\text{ MPa}$$

and  $S = 8.43$ . Therefore,  $\epsilon_d = 2.80\%$ . See Figure 67-4E.

Service load  $\epsilon_{50} = 2.99\%$

Live load  $\epsilon_{50} = 1.30\%$

Average Strains:

Service Load  $\epsilon_d = 2.80\%$

Live load  $\epsilon_L = 1.28\%$

Service Load  $\epsilon_{60} = 2.60\%$

Live load  $\epsilon_{60} = 1.25\%$

According to *LRFD* C14.7.5.3.3, service load deflection  $\Delta_d + \Delta_{\text{creep}}$  should not exceed 3 mm, as follows:

$$\Delta_d = \left[ \epsilon_d \left( n_{\text{layers}} h_{ri} + 2h_{re} \right) \right] = \frac{2.80}{100} [6(10 \text{ mm}) + 2(5 \text{ mm})]$$

$$\Delta_d = 1.96 \text{ mm}$$

Creep deflection to be taken as 30% of  $\Delta_d$  (*LRFD* Table 14.7.5.2-1) as follows:

$$\gamma_{\text{CR}} = 1.20 \text{ (LRFD Table 3.4.1-1)}$$

$$\Delta_{\text{creep}} = \gamma_{\text{CR}} 0.30 \Delta_d = (1.20) (0.30) (1.96 \text{ mm}) = 0.71 \text{ mm}$$

$$\text{Is } \Delta_d + \Delta_{\text{creep}} = 1.96 \text{ mm} + 0.71 \text{ mm} = 2.67 \text{ mm} < 3 \text{ mm?}$$

Check 3: OK

Live Load Deflection:

The compressive stress due to live loads is as follows:

$$\sigma_L = \frac{P_{\text{live}}}{LW} = \frac{(213.87 \text{ kN})1000}{(325 \text{ mm})(350 \text{ mm})} = 1.88 \text{ MPa}$$

Therefore,  $\epsilon_L = 1.28\%$  (see Figure 67-4E).

$$\Delta_{\text{Live}} = \epsilon_L \left( n_{\text{layers}} h_{ri} + 2h_{re} \right) = \frac{1.28}{100} [6(10 \text{ mm}) + 2(5 \text{ mm})]$$

$$\Delta_{\text{Live}} = 0.90 \text{ mm}$$

$$\Delta_{\text{Live}} + \Delta_{\text{creep}} = 0.90 \text{ mm} + 0.71 \text{ mm} = 1.61 \text{ mm}$$

The designer shall determine if deflection due to live load and pad creep provides acceptable rideability. See *LRFD* C.14.7.5.3.3.

According to *LRFD* 14.7.6.3.3, Initial Service Load Deflection,  $\Delta_d + \Delta_{live}$ , shall not exceed  $0.07 h_{ri}$ . For this pad,  $0.07 h_{ri} = (0.07)(10 \text{ mm}) = 0.70 \text{ mm}$ .

$$\Delta_d \text{ for one 10 mm layer} = \left( \frac{2.80}{100} \right) (10 \text{ mm}) = 0.28 \text{ mm}$$

$$\Delta_{live} \text{ for one 10 mm layer} = \left( \frac{1.28}{100} \right) (10 \text{ mm}) = 0.13 \text{ mm}$$

Total  $\Delta_d + \Delta_{live}$  for one 10-mm layer:  $\Delta_{di} + \Delta_{li} = 0.41 \text{ mm}$

Check 3A: OK

4. Check Elastomer Thickness based on Shear Deformation:

Load Factor  $\gamma_{TU} = 1.20$  for temperature effects at Service Limit State 1. See *LRFD* Table 3.4.1-1.

$$\Delta_s = \gamma_{TU} \alpha \Delta T L_e + \gamma_{CR} \Delta_{sc} = 1.20(0.0000117)(50)(11\,500 \text{ mm}) + 0$$

$$\Delta_s = 8.07 \text{ mm}$$

Is  $(h_{rt} > 2\Delta_s)$  satisfied? (*LRFD* Eq. 14.7.6.3.4-1)

$$h_{rt} = 70 \text{ mm} > 2\Delta_s = 16.15 \text{ mm? Check 4: OK}$$

5. Check Rotation (*LRFD* Eqs. 14.7.6.3.5-d1 and -d2):

Service load compressive stress due to total load is as follows:

$$\sigma_s = \frac{(P_{nd} + P_{cd} + P_{live} + P_{fws})}{LW} = \frac{(376.32 \text{ kN} + 31.39 \text{ kN} + 213.87 \text{ kN} + 66.91 \text{ kN})1000}{(325 \text{ mm})(350 \text{ mm})}$$

$$\sigma_s = 6.05 \text{ MPa}$$

Compare compression  $\sigma_s$  to AASHTO 14.7.6.3.5d. If the exterior layer thickness is greater than one-half the interior layer thickness, each exterior layer counts for one half of the interior layer.

$$0.5 G_{\max} S \left( \frac{L}{h_{ri}} \right)^2 \frac{\theta_{sx}}{n} = 0.5 (1.14 \text{ MPa}) (8.43) \left( \frac{325 \text{ mm}}{10 \text{ mm}} \right)^2 \frac{0.017 \text{ rad}}{6} = 14.37 \text{ MPa}$$

Is this  $< \sigma_s = 6.05 \text{ MPa}$  ? Check 5: NG

$$0.5 G_{\max} S \left( \frac{W}{h_{ri}} \right)^2 \frac{\theta_{sz}}{n} = 0.5 (1.14 \text{ MPa}) (8.43) \left( \frac{350 \text{ mm}}{10 \text{ mm}} \right)^2 \frac{0.005 \text{ rad}}{6} = 4.90 \text{ MPa}$$

Is this  $< \sigma_s = 6.05 \text{ MPa}$  ?      Check 6: OK

Since the first equation is not satisfied, either iterate the dimensions of the pad or use a tapered plate (shim) to match the grade (1%). In this instance, use a tapered plate, since the margin of error is large and reshaping of the pad probably will not satisfy this formula.

Therefore:

$$\text{grade} = 0.000 \text{ rad}$$

$$\theta_x = 0.002 \text{ rad}$$

$$\theta_{sx} = \theta_x + \text{grade} + 0.005 \text{ rad} = 0.007 \text{ rad}$$

$$\theta_{sz} = 0.005 \text{ rad}$$

$$0.5 G_{\max} S \left( \frac{L}{h_{ri}} \right)^2 \frac{\theta_{sx}}{n} = 0.5 (1.14 \text{ MPa}) (8.43) \left( \frac{325 \text{ mm}}{10 \text{ mm}} \right)^2 \frac{0.007 \text{ rad}}{6} = 5.92 \text{ MPa}$$

Is this  $< \sigma_s = 6.05 \text{ MPa}$  ?      Check 5: OK

$$0.5 G_{\max} S \left( \frac{W}{h_{ri}} \right)^2 \frac{\theta_{sz}}{n} = 0.5 (1.14 \text{ MPa}) (8.43) \left( \frac{350 \text{ rad}}{10 \text{ mm}} \right)^2 \frac{0.005 \text{ rad}}{6} = 4.90 \text{ MPa}$$

Is this  $< \sigma_s = 6.05 \text{ MPa}$  ?      Check 6: OK

6. Check Bearing Stability (*LRFD* 14.7.6.3.6):

$$L/3 > h_{\text{pad}}? \quad \frac{L}{3} = 108.33 \text{ mm} \quad \text{Check 7: OK}$$

$$h_{\text{pad}} = 87.50 \text{ mm}$$

$$W/3 > h_{\text{pad}}? \quad \frac{W}{3} = 116.67 \text{ mm} \quad \text{Check 8: OK}$$

7. Check Steel Reinforcement (*LRFD* 14.7.6.3.7 and *LRFD* 14.7.5.3.7):

*LRFD* 6.6.1.2.5 defines Constant Amplitude Fatigue Threshold as follows:

$$\Delta F_{TH} = 165 \text{ MPa for detail category A} \quad (\text{LRFD Table 6.6.1.2.5-3})$$

$$\text{Shim yield strength, } F_y = 250 \text{ MPa} \quad (\text{LRFD Table 6.4.1-1})$$

Compare  $h_s$  to *LRFD* Equations 14.7.5.3.7-1 and -2 as follows.

$$h_s > 3h_{r \max} \frac{\sigma_s}{F_y} \text{ and } 2h_{r \max} \frac{\sigma_L}{\Delta F_{TH}} ?$$

$$3h_{r \max} \frac{\sigma_s}{F_y} = 3 (10 \text{ mm}) \frac{6.05 \text{ MPa}}{250 \text{ MPa}} = 0.73 \text{ mm} \quad \text{Check 9: OK}$$

$$h_s = 2.50 \text{ mm}$$

$$2h_{r \max} \frac{\sigma_L}{\Delta F_{TH}} = 2 (10 \text{ mm}) \frac{1.88 \text{ MPa}}{165 \text{ MPa}} = 0.23 \text{ mm} \quad \text{Check 10: OK}$$

8. Determine whether or not mechanical anchorage is necessary (*LRFD* 14.7.6.4):

$$\text{Minimum vertical force anticipated } P_{sd} = \gamma_{pmin} (P_{nd} + P_{cd}) - \gamma_{LL} P_{uplift}$$

From *LRFD* Table 3.4.1-1 and 3.4.1-2, load factors for Strength I Limit State are as follows:

$$\gamma_{pmin} = 0.90, \gamma_{LL} = 1.75 \text{ and } \gamma_{TU} = 1.20$$

$$P_{sd} = 0.90 (376.32 \text{ kN} + 31.39 \text{ kN}) - 1.75 (14.19 \text{ kN}) = 342.11 \text{ kN}$$

Factored shear force at Strength Limit State I due to factored loads,  $H < (1/5)P_{sd}$ ?

$$H = G_{\max} A_{\text{pad}} \frac{\Delta_s}{h_{rt}} = 1.14 \text{ MPa} \frac{(113\,750 \text{ mm}^2)}{1000} \frac{8.07 \text{ mm}}{70 \text{ mm}} = 14.96 \text{ kN}$$

$$H_{\max} = \frac{P_{sd}}{5} = \frac{342.11 \text{ kN}}{5} = 68.42 \text{ kN} \quad \text{Check 11: OK}$$

Note: The maximum values of  $\gamma_{TU}$  and  $\gamma_{CR}$  of 1.20 are the same at both Strength I and Service I Limit States. Therefore, the value for  $\Delta_s$  determined in Design Check 4 can be used.

### Pad Summary

$$L = 325.00 \text{ mm} \quad W = 350.00 \text{ mm}$$

$$\text{Total Thickness: } h_{\text{pad}} = 87.50 \text{ mm}$$

$$\text{Number of interior elastomer layers: } n_{\text{layers}} = 6.00$$

Number of steel shims:  $n_{\text{shims}} = 7.00$

Deflection: Dead load only:  $\Delta_d = 196 \text{ mm}$

Live load only:  $\Delta_{\text{Live}} = 0.90 \text{ mm}$

Creep:  $\Delta_{\text{creep}} = 0.71 \text{ mm}$

## **67-4.06 Seismic Design**

### **67-4.06(01) Application**

The counties in Seismic Performance Zone 2 are Gibson, Posey, and Vanderburgh. All other counties are in Seismic Performance Zone 1. Each bridge therein should be designed in accordance with the criteria applicable to Zone 1. See Section 60-3.06 for more information.

### **67-4.06(02) Zone 1 Criteria**

The LRFD Specifications criteria for Zone 1 are as follows:

1. Minimum Support Length. Adequate support length is the most significant contributor to satisfactory performance of a bridge during a seismic event. The support length required by Article 4.7.4.4 of the *LRFD Specifications* shall be provided at the expansion ends of the structure unless longitudinal restrainers are provided.
2. Minimum Bearing Force Demands. The connection of the superstructure to the substructure shall be designed to resist a horizontal seismic force equal to 0.20 times (the dead load + 0.5 live load) reaction force in the restrained directions. See Article 3.10.9.2. No additional adjustment factors, loadings, or friction forces shall be applied to increase or decrease this minimum horizontal seismic force. This force shall not extend into the substructure design.

To preclude the need for restraining devices in Zone 1, expansion bearings shall resist the minimum horizontal seismic force in the transverse direction only. The minimum longitudinal seismic force resulting at an expansion bearing shall be applied at the nearest fixed bearing.

Fixed bearings, such as steel shoes, shall be attached to the pier cap with anchor bolts. Some examples of acceptable means of restraint at semi-fixed or expansion bearings in Zone 1 include

concrete shear keys between prestressed beams, beams resting in concrete channels, and steel side retainers bolted to the cap.

In designing the bearing connections for a Zone 2 bridge, the actual calculated seismic design forces, as adjusted by Article 3.10.7 of the *LRFD Specifications*, for Zone 2 shall be used. The longitudinal seismic forces at expansion bearings in Zone 2 may be resisted either by using seismic restraining devices (positive horizontal linkage), or they may be transferred to the bearing connections at the nearest fixed pier. Positive linkage shall be provided by means of ties, cables, dampers, or other equivalent mechanisms. Friction shall not be considered a positive linkage.

See Article 3.10.9.6 of the *LRFD Specifications* to determine if hold-down devices are required.

#### **67-4.06(03) Connections for Fixed Steel Shoes**

The connection between a fixed steel shoe and the pier cap shall be made with anchor bolts. The ultimate shear resistance in the anchor bolts, pintles, and high-strength bolts in the top shoe shall be verified that it is adequate to resist the calculated seismic forces of Seismic Performance Zones 1 and 2. See Article 6.13.2.7 of the *LRFD Specifications* and Figure 67-4G for determining the nominal shear resistance of anchor bolts and pintles. The minimum connections should be as shown in Figure 67-4G(1).

Masonry anchor bolts shall extend into the concrete a minimum of 380 mm. Anchor bolts used in Seismic Performance Zone 2 shall be in accordance with Article 14.8.3 of the *LRFD Specifications*.

Anchor bolts should be located beyond the limits of the bottom beam flange and interior diaphragm to ensure adequate clearance for anchor bolt installations and impact wrenches. The grade of structural steel used for the anchor bolts or pintles shall be shown on the plans.

Where the pintles can not be designed to accommodate the minimum seismic force of Zone 1, a hooded top shoe (see Figure 67-4H) shall be provided. A hooded top shoe may also be an acceptable seismic restrainer for a Zone 2 design. If seismic forces are extremely large, a restraining device will be required instead of the hooded shoe.

#### **67-4.06(04) Connections for Pot Bearings**

Fixed pot bearings shall meet the connection requirements for fixed steel shoes. The top bearing plate and lower masonry plate shall be bolted to the beam flange and the pier cap respectively.

Where welds are required between plates in the pot bearing, they shall be made continuous around the perimeter of the smaller plate.

#### **67-4.06(05) Connections for Elastomeric Bearings and PTFE/Elastomeric Bearings**

All elastomeric bearings and PTFE/elastomeric bearings shall be provided with adequate seismic-resistant anchorage to resist the transverse horizontal forces in excess of those accommodated by shear in the bearing. The restraint may be provided by one of the methods as follows:

1. steel side retainers with anchor bolts;
2. concrete shear keys placed in the top of the pier cap, or channel slots formed into the top of the cap or mudwall at the end bent (see Section 67-4.06(06)); or
3. concrete channels formed in the top of the end bent cap or expansion pier cap.

Steel side retainers and the anchor bolts shall be designed to resist the minimum transverse seismic force for the zone in which the bridge is located. The number of side retainers shall be as required to resist the seismic forces and shall be placed symmetrically with respect to the cross section of the bridge. Side retainers will often be required on each side of the girder flange of each beam line. The strength of the beams and diaphragms shall be sufficient to transmit the seismic forces from the superstructure to the bearings. A minimum of one 25-mm diameter anchor bolt shall be provided for each side retainer.

Concrete channels formed around each beam in the top of the end bent cap or expansion pier cap represent an acceptable alternative to steel side retainers. The top of the top shoe shall be set a minimum of 50 mm below the top of the concrete channel. If no top shoe is present, the bottom of the beam shall be placed 50 mm below the top of the channel. The minimum depth of the channel shall be 150 mm. The horizontal clearance from the side of the top shoe or edge of the beam to the side wall of the channel shall be 15 mm or less.

Integral end bents are a very effective way of accommodating the horizontal seismic forces of Zones 1 and 2. An integrally designed end bent will inherently resist the transverse seismic forces. Minimum support length requirements need not be checked for this type of substructure.

#### **67-4.06(06) Shear Keys at Semi-Fixed Prestressed I-Beam and Adjacent Box Beam Bearings**

Unreinforced shear keys should be provided between the beams at all semi-fixed supports. These shear keys typically rest in 300-mm wide by 900-mm long by 75-mm deep recessed keyways, the edges of which are also unreinforced. Although the shear keys are not structurally

designed, they are expected to adequately resist the horizontal seismic forces anticipated for either Zone 1 or 2.

To ensure that the shear keys will function as intended, keyways shall be provided between each beam line at every semi-fixed support, and an expanded polystyrene sheet, not exceeding 13 mm in thickness, shall be provided in the bottom of the keyway resulting in a minimum of 62 mm of shear key extending into the keyway.

Seismic restraint for an adjacent box beam bridge shall be provided with retaining blocks at the ends of the pier caps and end bent caps. These blocks shall be designed as reinforced shear keys and shall be in accordance with Article 5.8.4 of the *LRFD Specifications*.

#### **67-4.06(07) Seismic Isolation Bearings**

The use of seismic isolation bearings should be considered for a long-span steel bridge in Zone 2. The savings in substructure rehabilitation cost, resulting from an isolation bearing design, roughly offsets the substantial cost of the isolation bearings. The use of seismic isolation bearings should be based on performing a cost analysis comparing other alternatives, such as pot bearings or elastomeric bearings with suitable retainers or longitudinal restraining devices. The use of seismic isolation bearings in a Zone 1 structure does not appear to be cost effective.

The minimum bearing support length requirements of the *AASHTO LRFD Bridge Design Specifications* for seismic design shall be satisfied at the expansion ends of a bridge with seismic isolation bearings. The minimum bearing force demands should be assumed to be the actual calculated seismic forces of a Zone 2 isolation design.

Seismic isolation bearings significantly reduce the seismic forces on the substructure, possibly to the point where a non-seismic load case may control the pier design. This, however, does not relieve the designer of the need to provide pile anchorage, confinement steel in plastic hinge regions, and proper location of lap splices in designing a Zone 2 structure. The design of seismic isolation bearings shall be in accordance with the *AASHTO Guide Specifications for Seismic Isolation Design*. The *LRFD Bridge Design Specifications* requires that each bearing system shall be tested under both static and cyclic loading prior to acceptance. The designer shall prepare a unique special provision which includes the testing requirements.

#### **67-4.07 Bearing Assembly Details**

The following figures provide typical details for acceptable connections for bearing assemblies.

1. Fixed Shoe Assembly. See Figure 67-4H.

2. Elastomeric Bearing Assembly. See Figures 67-4I, 67-4J, and 67-4K.
3. PTFE/Elastomeric Bearing Assembly. See Figure 67-4L.

These are suggested details. They may be revised as necessary for each project. Also, see the INDOT *Standard Drawings* for more bearing details.

## **67-5.0 BRIDGE SEAT ELEVATIONS**

### **67-5.01 General**

In establishing bridge seat elevations at both end and interior supports, the following should be considered:

1. bridge deck depth;
- 2.\* 20-mm fillet (see Section 61-4.02(02) for steel beams/girders and 61-4.02(03) for concrete beams);
- 3.\*\* residual beam camber, or difference between design camber and dead load deflection of slab. At interior supports, use the larger value from adjacent spans;
- 4.\*\* vertical curve effect: (+) for sag vertical curve, (-) for crest vertical curve;
5. beam depth;
- 6.\*\* beam notch; and
7. bearing thickness, including shims.

\* *The fillet distance is measured from bottom of deck to top of beam. This distance is included to allow for variation in beam camber.*

\*\* *Precast concrete beams only.*

The accuracy for establishing bridge seat elevations shall be to the nearest millimeter.

### **67-5.02 Examples**

The following figures and examples provide information for determining bridge seat elevations.

Figure 67-5A Typical Bridge Section (Concrete I-Beam Example)

Figure 67-5B	Fillet Dimensions for Concrete I-Beams
Figure 67-5C	Seat Elevations for Concrete I-Beam Example
Figure 67-5D	Typical Bridge Section (Built-Up Plate Girder Example)
Figure 67-5E	Seat Elevations (Steel Beam or Plate Girder)
Figure 67-5F	Fillet Dimensions for Steel Beams
Figure 67-5G	Seat Elevations for Steel Beam or Plate Girder Example